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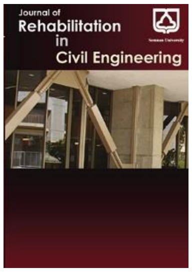

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Comparison of Progressive Collapse Capacity of Steel Moment Resisting Frames and Dual Systems with Buckling Retrained Braces

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ABSTRACT

Progressive collapse refers to a condition where local failure of a primary structural component leads to the collapse of neighboring members and the whole structure, consequently. In the present study, the progressive collapse potential of seismically designed steel dual systems with buckling restrained braces is inquired applying the alternate path method, and their performances are compared with those of the conventional intermediate moment resisting frames. Static nonlinear Push-down and dynamic analyses under gravity loads specified in GSA guideline are conducted to capture the progressive collapse response of the structures as a result to the column and adjacent BRBs removal, and their ability of absorbing the destructive effects of member loss is investigated. It was observed that, compared with the intermediate moment resisting frames, generally the dual systems with buckling restrained braces provided appropriate alternative path for redistributing the generated loads caused by member loss and the results varied more significantly depending on the variables such as location of column loss, or number of stories. Moreover, in the most column removal scenarios, steel dual systems are more capable to resist the progressive collapse loads and maintain the structural overall integrity.

1. Introduction

Dual systems with Buckling Restrained Braces (BRBs) are primarily employed as lateral load resisting systems in the structures located in seismic-active areas. These systems consist of buckling restrained braces arranged in various configurations, such as diagonal, Inverted-V (chevron), Double-story X, etc. in conformity with their heights and have the advantages of both moment resisting frames and buckling restrained braced frames. The main characteristics of BRBs are the high ductility, excellent energy

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dissipation, and nearly symmetrical hysteretic response in tension and compression. The core segment of a BRB is filled with unbounded concrete (mortar) that offers resistance to both in-plane and out-of-plane buckling of the steel plate inside, thereby boosting the axial resistance and energy dissipation potential of BRBs under the compressive axial loading [1]. The modern buckling restrained braces are called all-steel BRBs, are entirely made up of steel components. The buckling restraining mechanism in these type of BRBs are usually steel hollow structural members as an alternative for conventional mortar or concrete filled steel tubes.

Progressive collapse is known as the failure of all or a significant part of a structure triggered by damage or failure of a relatively small part of the structure [2]. A progressive collapse can be initiated by causes such as design and construction errors and load events which are not deliberated by the structural engineer [3]. These so-called abnormal loads are outside the normal structural design basis. As a historical perspective, the collapse of the Ronan Point apartment building in London on 1968, was one of the first recorded episodes of progressive collapse [4]. Contemplating the collapse of the Ronan Point apartment, the progressive collapse has been a major design consideration. Recently, interest in this topic has also increased as a result to the terrorist attacks on the Alfred P. Murrah building in Oklahama City in 1995 and the World Trade Center in New York in 2001 [5].

Different codes and guidelines have inquired the progressive collapse and have provided several solutions in order to design the structures against its destructive effects. The Progressive collapse analysis and design guidelines provided by General Services Administration (GSA) [6] and the Unified Facilities Criteria (UFC) [7] allocated by Department of Defense (DoD), are two existing progressive collapse design guidelines. These two guidelines employ the Alternate Path Method (APM) to evaluate a structural system in order to compute its vulnerability to progressive collapse. The alternate path approach presumes that one critical or key member, typically a column, is damaged and rendered incapable of supporting load [8]. The analyzing procedures for the alternate path method include both static and dynamic analyses. However, the key issue in progressive collapse is to deliberate that it is a dynamic event [9] and the load redistribution effects will arise dynamically during the local collapse. Therefore, pondering the dynamic effects are essential in evaluation of progressive collapse potential of structures.

Min Liu [10] applied genetic algorithm to cost effectively design of seismic two-dimensional steel moment frames and subsequently assessed the progressive collapse potential of these frames employing the alternate path method. It is been found that the structures with optimal weight design in which the seismic design guidelines which are considered, are more vulnerable to progressive collapse. Khandelwal et al. [11] inspected the progressive collapse resistance of seismically designed steel braced frames. Two types of braced systems were considered, namely, special Concentrically Braced Frames (SCBF) and Eccentrically Braced Frames (EBF). The results revealed that while both systems benefited from locating the seismic systems on the perimeter of the buildings, the EBF designed for high seismic risk is less vulnerable to gravity-induced progressive collapse than the SCBF designed for moderate seismic risk. Jinkoo Kim and Taewan Kim [12] assessed the progressive collapse resisting capacity of steel moment frames. They found that nonlinear dynamic analysis provides larger structural responses and the results vary more significantly depending on the variables such as applied load, location of column removal, or number of building stories. Jinkoo Kim et al. [13] examined the progressive collapse resisting capacity of braced frames by performing nonlinear static and dynamic analyses. Pursuant to the results from dynamic analyses, it was found that the model structures generally remained stable after the first story central column was suddenly removed. Nonlinear static push-down analysis results displayed that the model structures had inherent strength twice as high as the strength required by the GSA guideline. Exceptionally, the K-braced frame in which premature failure occurred due to column buckling.

Tavakoli and Kiakojouri [14] assessed the influence of sudden column loss on dynamic response of steel moment frames under blast loading. In their work, progressive collapse capacity of steel moment frames was inquired applying alternate load path method. Nonlinear dynamic analysis was carried out to examine the response of the steel moment frames in blast and sudden column loss scenarios. According to the results, progressive collapse potential is strongly dependent on location of column loss. The effect of local damage on energy absorption of steel frame buildings during earthquake was inspected by Parsaeifard and Nateghi [15]. The results manifested that collapse pattern is in a way that the damaged frame as well as the nearby frames has the most participation in supporting lateral deformations, and by distancing away from the damaged frame, deformation of the frames decreases.

Chen et al. [16] developed a probabilistic assessment method of a steel framed building under abrupt removal of a column as a result to catastrophic events. A robustness index (RI) was proposed to evaluate the structural robustness performance based on the acceptable probability of global failure and structural collapse probability.

Mashhadi and Saffari [17] investigated the effect of post-elastic stiffness ratio of members on Dynamic Increase Factor (DIF) in nonlinear static analysis of structures against column removal and a modified empirical DIF was proposed. The results of analysis revealed that the post-elastic stiffness ratio significantly affects the DIF. Therefore, the new empirical formulas including moment demand, ductility and post-elastic stiffness ratio were suggested.

Zhong et al. [18] persuaded the anti-progressive collapse performance of different stiffened connections of a steel frame. It was found that if the constraint provided by the side column is sufficient to develop catenary action in a beam, the constraint provided by peripheral components will have little effect on the bearing capacity against progressive collapse. Salmasi and Sheidaii [19] assessed the strength of dual steel moment frames equipped with a variety of eccentric bracings against progressive collapse by employingnonlinear static alternate path method. The results displayed that dual steel moment frames equipped with eccentric bracings generally exhibited desirable strength against progressive collapse and alternate in the type of bracing resulted in significant changes in the system capacity in the progressive collapse. The relationship between the seismic design parameters of a building and its progressive collapse-resisting capacity are the topic of recent researches in this area. Nevertheless, most of prior works have been focused on the progressive collapse response of moment resisting frames. In addition, a number of studies have inquired the progressive collapse behavior of braced frames as well. The ability of applying steel braces in order to reduce the potential of progressive collapse has been investigated by Bandyopadhyay and Banik [20] and the results exhibited that the braced frames are less prone to progressive collapse as a result to catenary action of the braces inside the frame, after column removal.

The progressive collapse behavior of modern seismic resistant systems such as buckling restrained braced frames and moment resisting frames equipped with buckling restrained braces (dual systems) have not been inquired broadly, while these systems are widely being applied all over the world.

This paper aims to investigate the influence of catenary action of buckling restrained braces on reduction of progressive collapse potential of steel moment resisting frames. For this purpose, 4 and 10 story moment resisting building frames and dual systems with buckling restrained braced frames are deliberated for the static and dynamic analysis.

2. Analysis Procedure

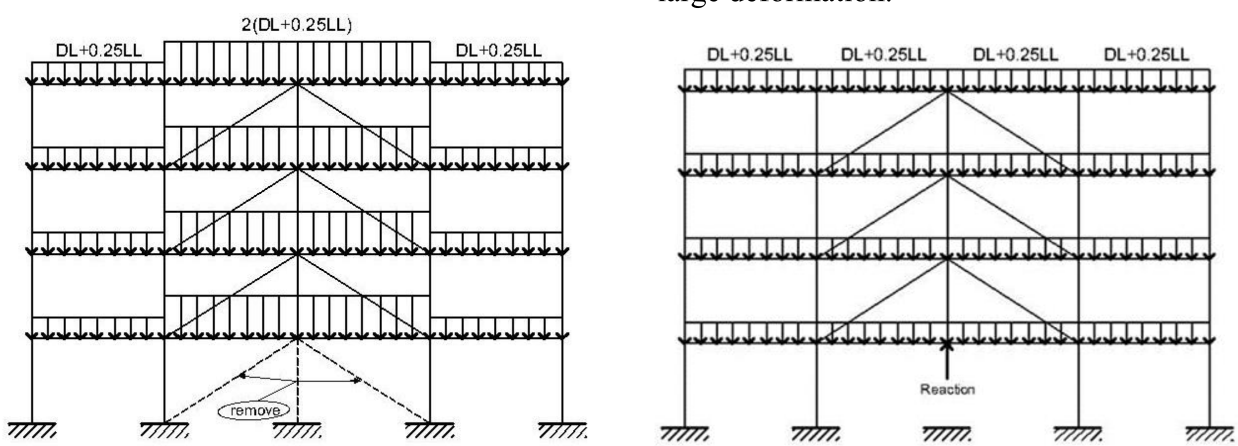
Among different design methods against the progressive collapse, the guidelines typically recommend the alternate path method. In this method, the elimination of a key and critical element is being examined and the structures are then analyzed with the purpose of detecting the consequent effects. When a structural element is removed abruptly, the rest of the structure should be able to tolerate the redistributed loads for a certain period of time.

The guidelines commonly endorse the following analysis procedures for the alternate path method: Linear Static (LS), Linear Dynamic (LD), Nonlinear Static (NS), and Nonlinear Dynamic (ND) methods. Since the nonlinear procedures are more accurate in contrast to the linear ones, nonlinear analysis procedures have been employedin the present study.

As a whole, in a nonlinear analysis, two kinds of nonlinearity can be considered. One of them is the geometric nonlinearity which is related to P-Delta effects and the other is material nonlinearities. The P-Delta effect is deliberated to take into account the effect of gravity loads on the lateral stiffness of the structures. This is underlined in most of building design codes. This effect is contemplated in the present study as the geometric nonlinearity. In order to apply the material nonlinearity, fiber element models are defined and assigned to the elements. Each fiber is characterized by the respective material relationship.

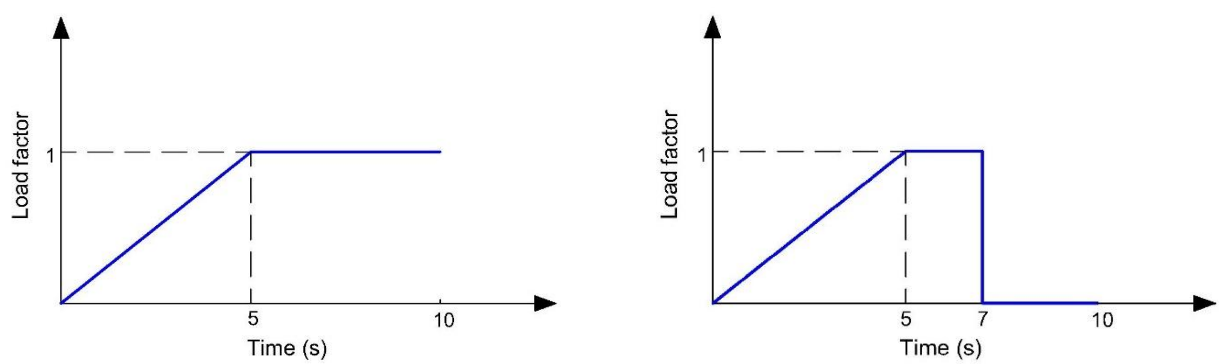
Nonlinear static analyses are performed after removing the critical elements from the structural model. In each analysis, only one critical element is removed. Fig. 1a portrayed the imposed loading pattern for progressive collapse in static analyses procedure. As illustarted in this Fig. 1a, all the structural bays are loaded by (Dead Load+0.25Live Load) except the bay which is associated directly with the removed column. This bay is loaded by 2(Dead Load+0.25Live Load). Pursuant to the GSA 2003 guidelines, the dynamic increase factor 2 is utilized to apply the dynamic effects of the progressive collapse in the static procedure.

Nonlinear dynamic analyses are performed by removing one critical column and adjacent BRB member in dual system. In the moment resisting frame system, the progressive collapse analysis is conducted by only one critical column removal. The column and the adjacent BRBs are abruptly removed at the design load level and the dynamic response of the structure is identified. Fig. 1b indicates the imposed loads for progressive collapse in dynamic analyses. The time history functions which have been employed in dynamic analysis are depicted in Fig. 2. In this study the forces were increased linearly for five seconds until they reached their full amounts, kept unchanged for two seconds until the system reached stable condition, and the upward force was suddenly removed at seventh second to simulate the dynamic effect caused by sudden removal of the column. For solving the equilibrium equation of motion, the Hilber-Hughes-Taylor numerical time-step method is applied. In all of the solution algorithms, the time step size must be selected significantly smaller than the time interval of the column removal [21]. In addition, the Rayleigh's damping is applied in all models. Damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformation.



(a) Static Analysis (b) Dynamic Analysis

Fig. 1. Imposed loads for progressive collapse analysis.



(a) Static Analysis (b) Dynamic Analysis

Fig. 2. Time histories of imposed loads for dynamic analysis.

3. Description of Analytical Models

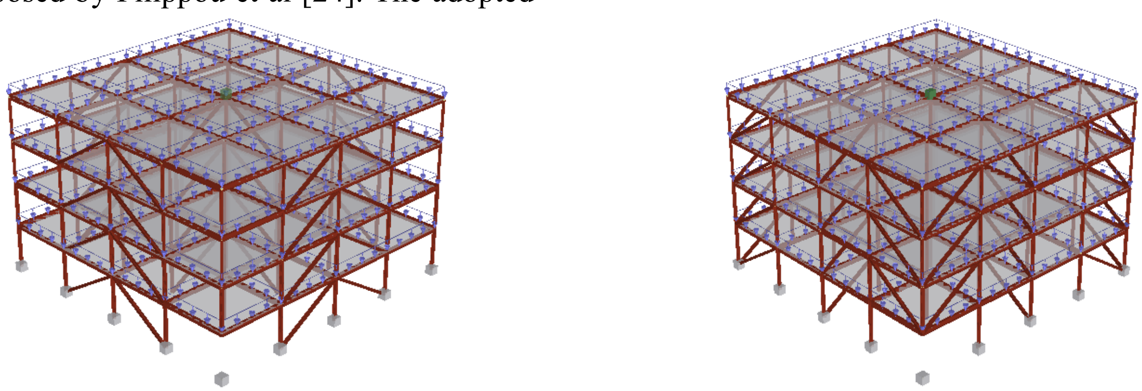
In the current study, the vulnerability of six different low (i.e. 4 story) and mid-rise (i.e. 10 story) residential buildings against progressive collapse is inquired by applying 3D nonlinear static and dynamic analysis employing the SeismoStruct [22] software. In order to recognize the effect of lateral load bearing system of the buildings, two different seismic load resisting systems are used: the dual system with diagonal buckling restrained braces and Intermediate Moment-resisting Frames (IMFs). Fig. 3 represents the 3D model of 4 story dual system with two bracing patterns.

For all buildings, the constitutive model employed for the St37 steel was the one proposed by Menegotto and Pinto [23] coupled with the isotropic hardening rules proposed by Filippou et al [24]. The adopted material properties include the Young's modulus of 210 GPa, Poisson ratio of 0.3, and density of 7850 kg/m3. The yield and ultimate stresses of the steel material are assumed as 240 and 370 MPa, respectively. All buildings have a uniform story height of 3.2 m. The plan dimensions of the buildings are shown in Fig. 4. The following assumptions are deliberated in this study:

-All structural models are designed to resist both gravity and lateral loads as a result to strong earthquakes pursuant to Iranian building codes [25] applying ETABS 2015 software.

- Design of steel elements and connections are based on AISC 360 [26] regulations.

- Progressive collapse analyses are conducted according to GSA 2003 regulations in SeismoStruct 2016 software.



(a) Dual system, config. 1

(b) Dual system, config. 2

Fig. 3. FEM model of 4 story dual system in SeismoStruct.

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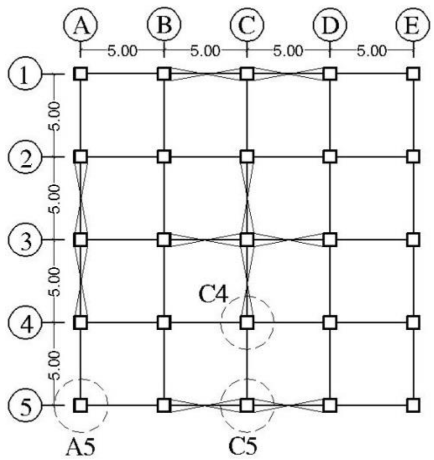
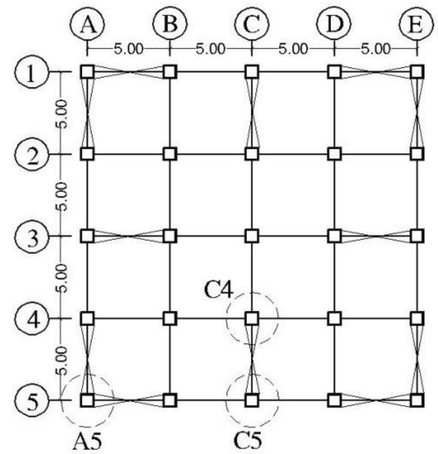
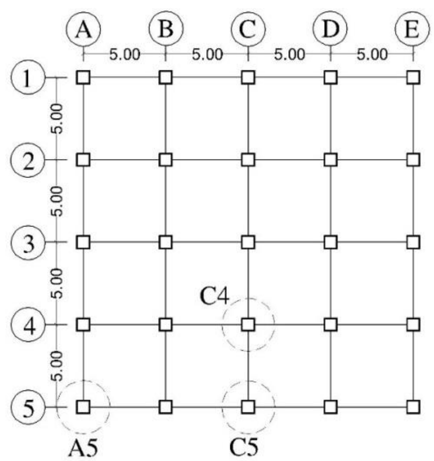


(a) Dual System, configuration1



(b) Dual System, configuration 2



(c) IMF

Fig. 4. Typical plan of model structures.

The design dead and live loads for the surrounding walls and floor areas are indicated in Table 1. In addition, column removal cases are shown in Table 2. The buildings are assumed to be located at a high seismic zone in Iran.

Nodal constraints with a penalty function option were adopted in order to model the rigid diaphragm effect. The penalty function exponent used was set to 1010. The weight of each floor was assumed to be lumped in the floor nodes, pursuant to the respective tributary area. An initial mid span imperfection was applied for all braces and columns to capture the post-buckling behavior of elements. Geometric properties of beams and columns are illustrated in Table 3. In addition, the characteristics of BRBs in dual systems are displayed in Table 4. Based on the design data, the amount of steel used in 4 and 10 story dual systems is evaluated as 72 tons and 284 tons, respectively. The corresponding values for the moment resisting frames are 90 tons and 316 tons for 4 and 10 story buildings, respectively.

Table1. Design gravity loads in model structures.

Story	Deck Dead Load (KN/m^2)	Deck Live load (KN/m^2)
All stories except roof	4.70	2.50
Roof story	5.70	1.50

Table 2. Column removal scenarios.

Case (Loss scenario)	**Story**	**column**	**Removed Element(s)**
1	1st and mid-height* story	Corner	A5 Column and adjacent BRBs(in dual system)
2	1st and mid-height* story	middle	C5 Column and adjacent BRBs(in dual system)
3	1st and mid-height* story	interior	C4 Column and adjacent BRBs(in dual system)

*The third story in four story building and the fifth story in ten story building

Table 3. Member sizes of model structures.

Number of stories	**Load-resisting system**		**Column section (cm)**	**Beam section**
4 story	Dual, configuration 1& 2	Story 1 & 2	Box 20×20×1.5	W 14×34
		Story 3 & 4	Box 15×15×1	W 14×26
	IMF	Story 1 & 2	Box 30×30×2	W 16×45
		Story 3 & 4	Box 25×25×1.2	W 16×36
10 story	Dual, configuration 1&2	Story 1 & 2	Box 45×45×2.5	W 16×40
		Story 3 & 4	Box 40×40×2	W 16×40
		Story 5 & 6	Box 35×35×1.5	W 16×40
		Story 7 & 8	Box 25×25×1	W 16×40
		Story 9 & 10	Box 20×20×1	W 16×40
	IMF	Story 1 & 2	Box 45×45×2	W 21×62
		Story 3 & 4	Box 40×40×2	W 21×62
		Story 5 &6	Box 35×35×2	W 21×62
		Story 7 & 8	Box 30×30×1.5	W 18×50
		Story 9 & 10	Box 25×25×1.2	W 18×50

Table 4. Geometric Properties of BRBs in dual system.

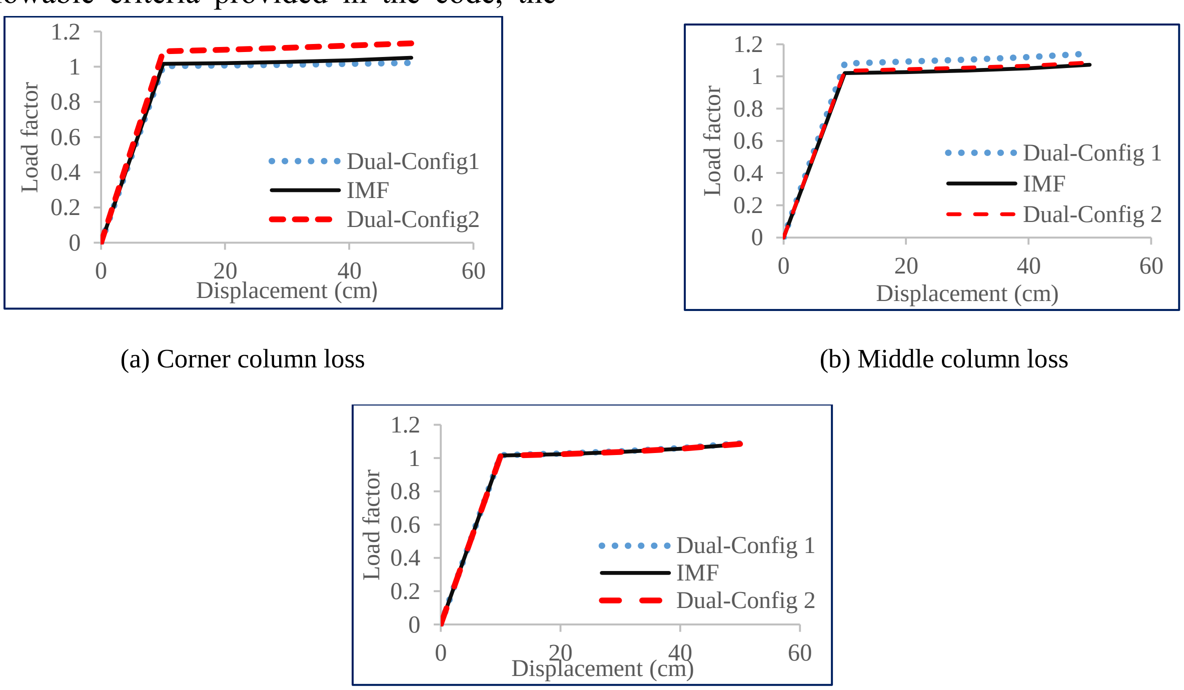
Number of stories	**Load-resisting system**	**Story**	**Area (mm^2)**
4 story	Dual, configuration 1& 2	Story 1 & 2	2200
		Story 3 & 4	1600
10 story	Dual, configuration 1&2	Story 1 & 2	5000
		Story 3 & 4	4200
		Story 5 & 6	3750
		Story 7 & 8	2500
		Story 9 & 10	1600

4. Analysis Results

4.1. Static Push down Analysis

In order to carry out nonlinear static push-down analysis, first the candidate column is detached from the structural model, and the displacement of the top joint of the removed column is gradually increased. At every step during the push-down analysis, the ratio of the applied load and the GSA-specified load combination called the 'load factor', is captured. The "load factor"-displacement diagram is computed by the push-down analysis. If the maximum load factor in the diagram is less than 1.0, it means that the structure is not able to appropriately resist the progressive collapse loads, and exhibits a high potential for progressive collapse. Nevertheless, if the maximum load factor is greater than 1.0, and the member rotation and ductility do not exceed the maximum allowable criteria provided in the code, the structure will be deliberated as a progressive collapse resistant system.

Nonlinear push-down analysis results of 4 story dual system, for corner, middle, and interior column loss cases, are indicated in Fig. 5. As it can be observed in Fig. 5, the maximum value of load factor is greater than 1.0 for all column removal cases. The higher load factors reveal that after column removal, the rest of the structures can absorb the column loss and alternate path is devoted to redistribute the loads. In addition, while a corner or a middle column in the first story is detached, dual systems with both configurations are able to assign better alternative path in comparison to IMFs (Fig. 5b). Captured load factors for all systems are the same in case of interior column removal as portrayed in Fig. 5c. As can be observed in Fig. 6, by removing column in the third story, the results of both systems for all scenarios of column loss are almost the same.

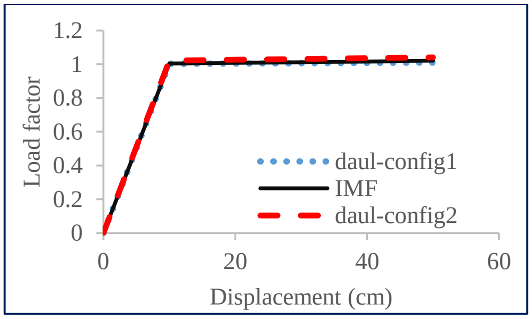


(a) Corner column loss

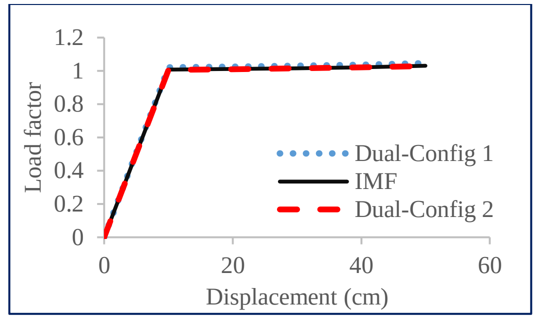
(b) Middle column loss

(c) Interior column loss

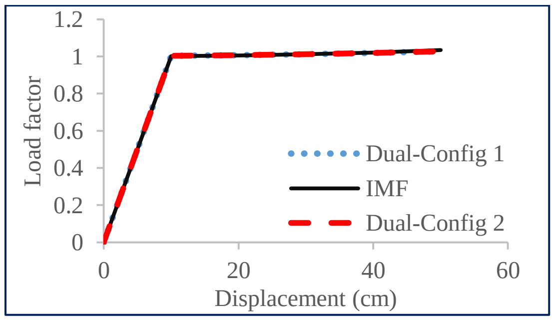
Fig. 5. Load-displacement diagram of the 4 story frame for 1st story column loss.



(a) Corner column loss



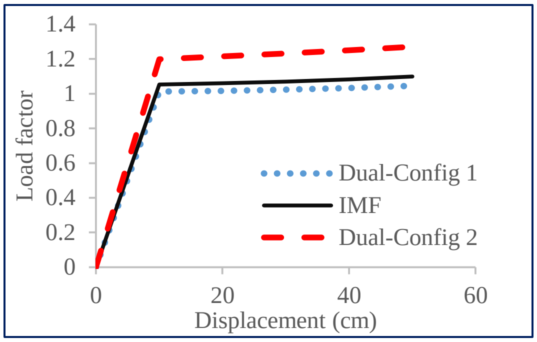
(b) Middle column loss



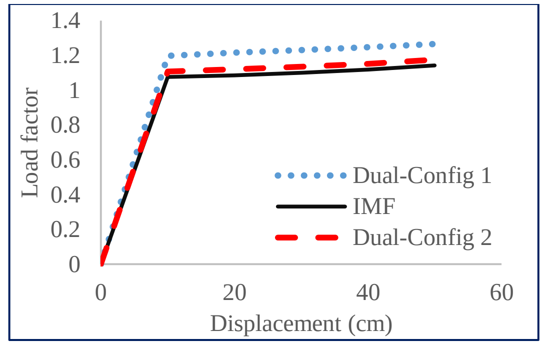
(c) Interior column loss

Fig. 6. Load-displacement diagram of the 4 story building frame in case of third story column loss.

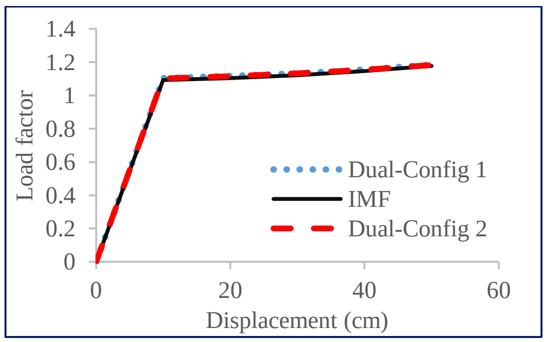
Deliberating the 10 story dual and intermediate moment resisting frame systems and performing nonlinear push-down analysis, the maximum reported load factor values are greater than 1.0 in cases of removing the corner, middle and interior column as illustrated in Figs. 7 and 8. Though, the value of these factors increases by increasing the number of building stories. It means that, as the building becomes taller, its progressive collapse resisting capacity rises, due to the fact that the number of elements participate in load carrying after the column removal increases significantly. Furthermore, when a corner column in the first story is detached, dual system with configuration 2 has greater load factors in comparison to IMF and the load factors of IMF are also higher than those in the dual system with configuration 1. In fact, in dual system with configuration 1, the beam elements with smaller sizes in comparison to those in IMF, which are directly associated with the removed column, do not have the required strength to withstand the progressive collapse loads and some plastic hinges were formed in the members (Fig. 7a). Once middle column was removed, dual system with two configurations have greater load factors in comparison to IMF (Fig. 7b).



(a) Corner column loss



(b) Middle column loss

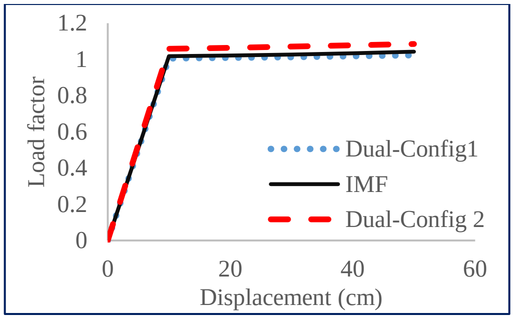


(c) Interior column loss

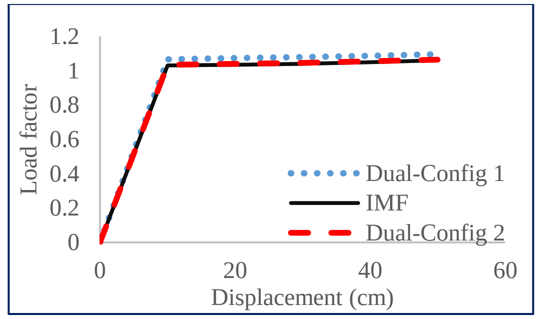
Fig. 7. Load-displacement diagram of the 10 story frame for 1st story column loss.

It can be stated that when a middle column was detached, the catenary action of beams surrounding the column is more highlighted as a result to the presence of two BRB members above the removed column. During the interior column removal, all structural systems have the same load factors (Fig. 7c). Fig. 8 manifests that when column loss takes place in the fifth story, the load factors decreases; despite that, they still remain above 1.0. Moreover, the structural systems in all scenarios of column removal have almost the same load factors.

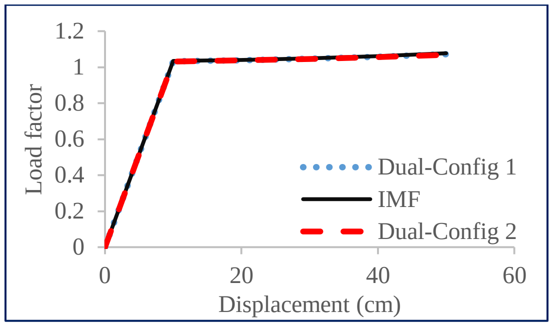
The comparison of maximum strength in dual systems for buildings with different heights is depicted in Figs. 9 and 10. As it is evident in Figs. 9 and 10, the progressive collapse resisting capacity increases as the number of building story increases in all of the dual structural models. As it is displayed in Fig. 8, the buildings with dual lateral load resisting systems have no progressive collapse potential for the removal of any column and significantly depending on location of column removal. Moreover, Figs. 9 and 10 persuade that the load factors decrease by increase in story of column loss. Thus, the dual system with configuration 2 exhibits a better performance against progressive collapse when the corner column was detached. During the middle and interior column removal, the dual system with configuration 1 has a better performance indicating that more suitable alternative path will be supplied to withstand the redistributed forces via the BRBs, as shown in Figs. 9 and 10.



(a) Corner column loss

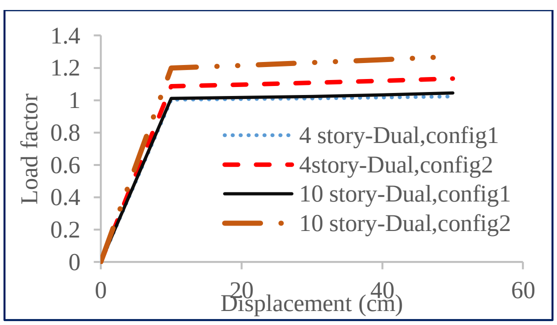


(b) Middle column loss

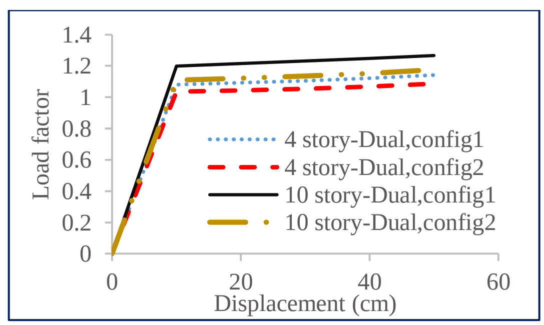


(c) Interior column loss

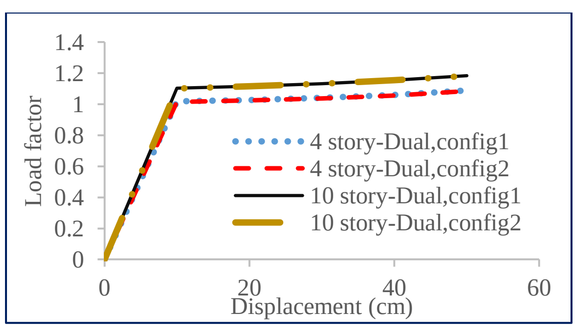
Fig. 8. Load-displacement diagram of the 10 story frame for 5th story column loss.



(a) Corner column loss

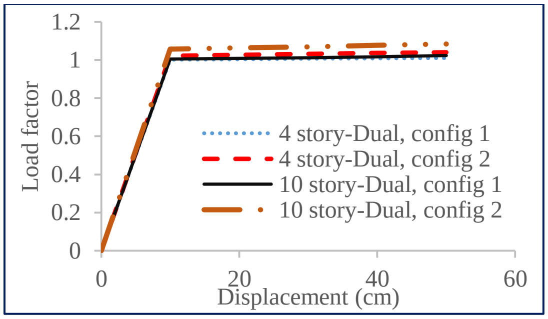


(b) Middle column loss

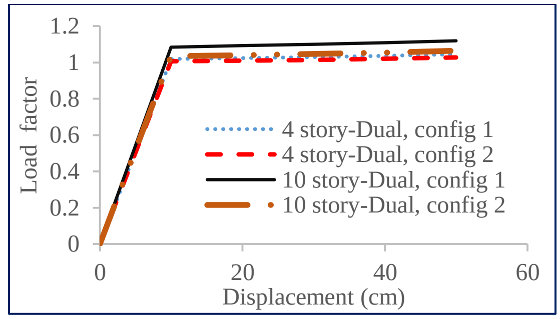


c) Interior column loss

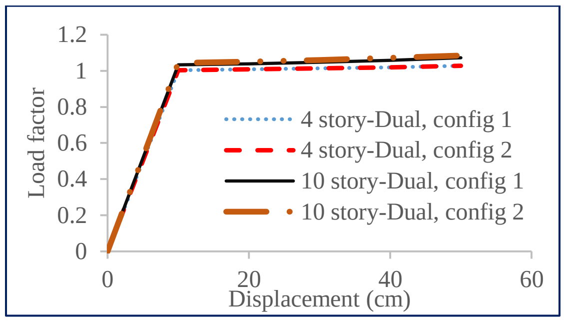
Fig. 9. Comparison of maximum strength in dual systems for 1st story column loss.



(a) Corner column loss



(b) Middle column loss



c) Interior column loss

Fig. 10. Comparison of maximum strength in dual system for mid-height story column loss.

4.2. Nonlinear Dynamic Analysis

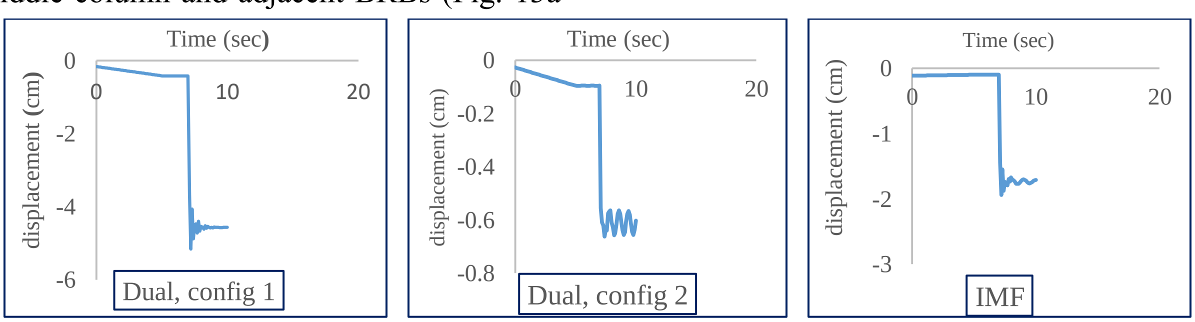
Nonlinear dynamic analyses are carried out in order to compute the structural response against the sudden column loss. Time histories of imposed dynamic loads are portrayed in Fig. 2. As the progressive collapse load increases linearly, the removed column reactions increase linearly as well. When these loads reach their maximum value, the reactions remain unchanged for a few seconds until the structure reaches a stable condition. Then, the removed column reactions decrease to zero abruptly to simulate the dynamic effects caused by the sudden column loss. The duration of elimination must be less than one tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column, as specified in UFC [27].

Nonlinear dynamic analyses are conducted and the results are summarized in the form of time-displacement history diagrams in Figs. 11 to 14. The nonlinear dynamic analysis results indicate that in the IMFs and dual systems, the vertical displacement of the top joint, from which the column has been detached, boosts abruptly yet this increase is not great enough to make large rotation in the elements. Then, this joint vibrates around a static equilibrium position and finally stops when the vibration amplitude dissipates. Moreover, Figs. 11 to 14 represent the displacement time history at the point of the column removal in time duration of 10 seconds under gravity load that specified in GSA 2003.

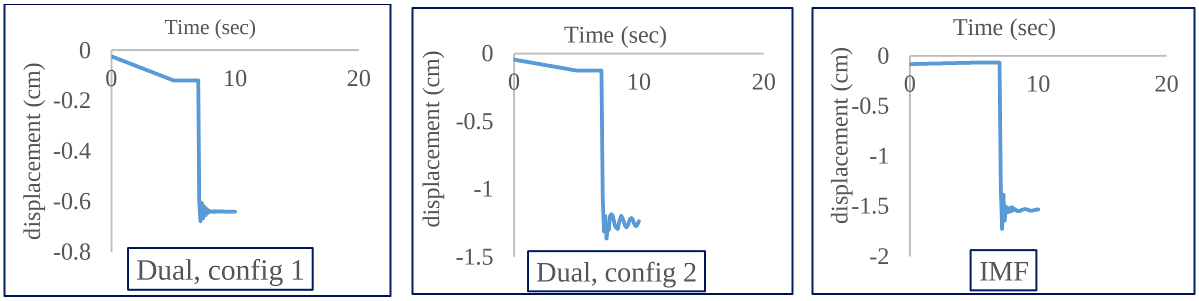
As highlighted in Fig. 11, the peak vertical displacements of the nodes corresponding to the top of the removed columns in the 4 story building are 4.5 cm, 0.62 cm, and 1.75 cm for dual system with configuration 1, dual system with configuration 2, and IMF, respectively, when the corner column and adjacent BRBs are removed. In case of middle column loss, the corresponding peak vertical displacements are 0.64 cm, 1.2 cm, and 1.55, respectively. In the last case, when the interior column and adjacent BRBs are detached, the peak vertical displacements of the nodes are 1.4 cm, 1.4 cm, and 1.52 for dual system with configuration 1, dual system with configuration 2, and IMF, respectively.

Fig. 12 exhibits that the peak vertical displacements of the nodes corresponding to the top of the removed columns in 10 story building are 1.8 cm, 0.45 cm, and 1.1 cm for dual system with configuration 1, dual system with configuration 2, and IMF, respectively, when the corner column and adjacent BRBs are removed. In case of middle column loss, the peak vertical displacements of the nodes are 0.45 cm, 0.85 cm, and 1cm, respectively. In the last case, when the interior column and adjacent BRBs are removed, the corresponding displacements are 0.86 cm, 0.86 cm, and 1.03, respectively.

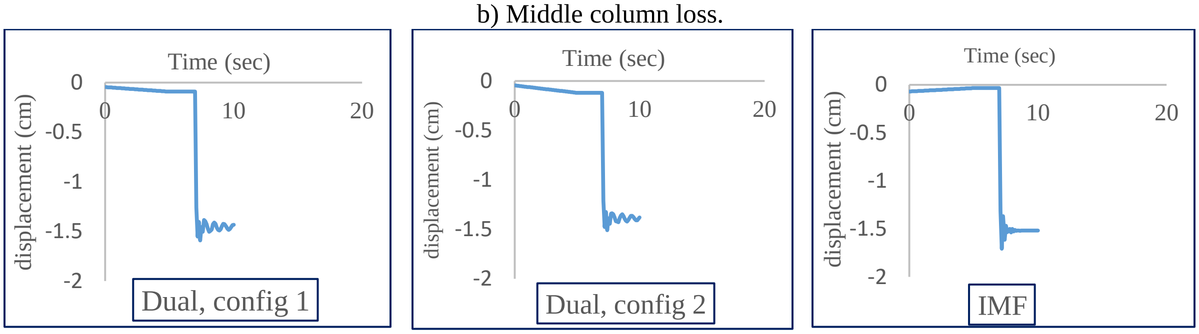
Figs. 13 and 14 depict that vertical displacement at the top point during the mid-height column removal is almost greater than that during the first story column removal case. In the case of removing the corner and middle column and adjacent BRBs (Fig. 13a and 13b), the vertical displacement of the dual system with configuration 2 is less than that in IMF. In addition, in loss scenario 1, the vertical displacement of dual system with configuration 1 (11cm) is higher than that in IMF (i.e. 2.25cm). In loss scenario 3 or case 3 as illustrated in Fig. 13c, the vertical displacement of IMF is less than that in dual systems with configuration 1 and 2, which can be associated to the smaller beam and column sizes of dual systems in comparison to IMFs. As displayed in Figs. 14a and 14b, after removing the fifth story column and adjacent BRBs in 10-story building, vertical displacement of the dual systems with configuration 1and 2 is less than that in IMF, except in case 1, in which the vertical displacement of IMF is less than that in dual system with configuration 1. Moreover, Fig. 14c reveals that in loss scenario 3, the dual systems with configurations 1 and 2, and also IMFs, experience the same vertical displacements. Nonlinear dynamic analysis results indicate that the vertical displacements of the nodes corresponding to the top of removed columns for corner column loss is greater than those in middle and interior column loss cases. Based on the analysis results, the progressive collapse response of the systems significantly depends on the location of critical element(s) loss.



(a) Corner column loss.



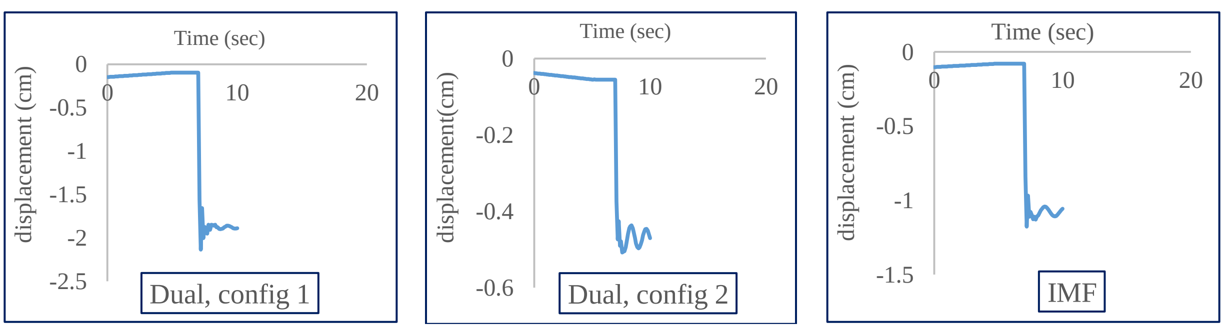
b) Middle column loss.



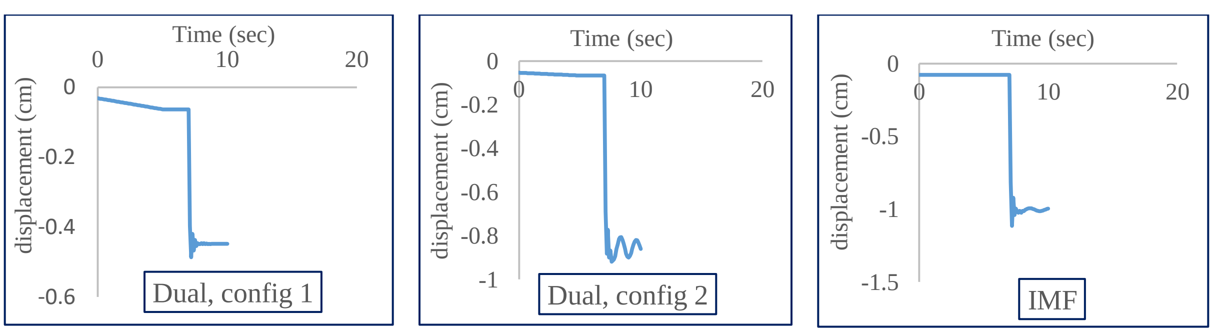
(c) Interior column loss.

Fig. 11. Nonlinear dynamic analysis results of 4 story buildings; 1st story column loss.

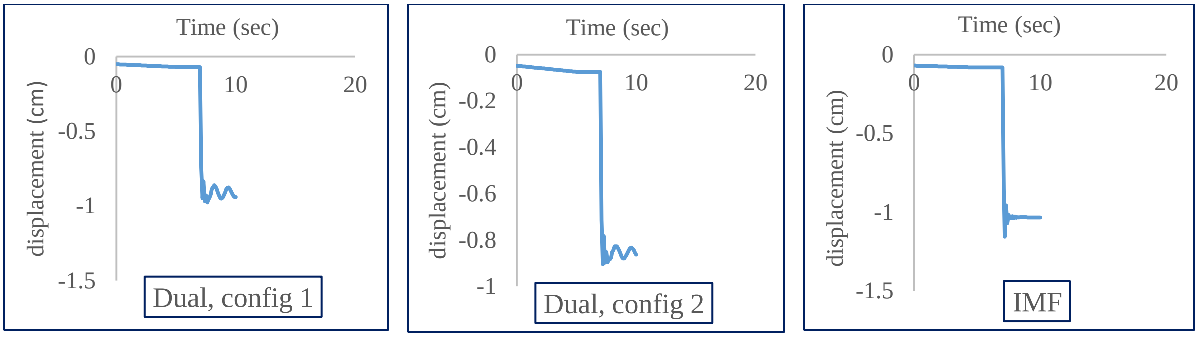
The progressive collapse resistance of dual system is higher than IMF especially when the BRBs are devoted above the column loss point. In the other words, the progressive collapse response of dual system is expressively depending on the placement of BRBs and also the location of column loss.



(a) Corner column loss.

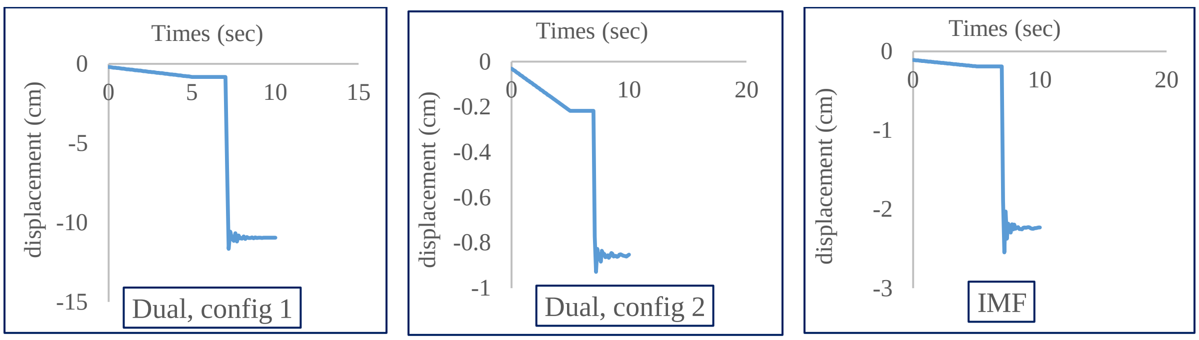


(b) Middle column loss.

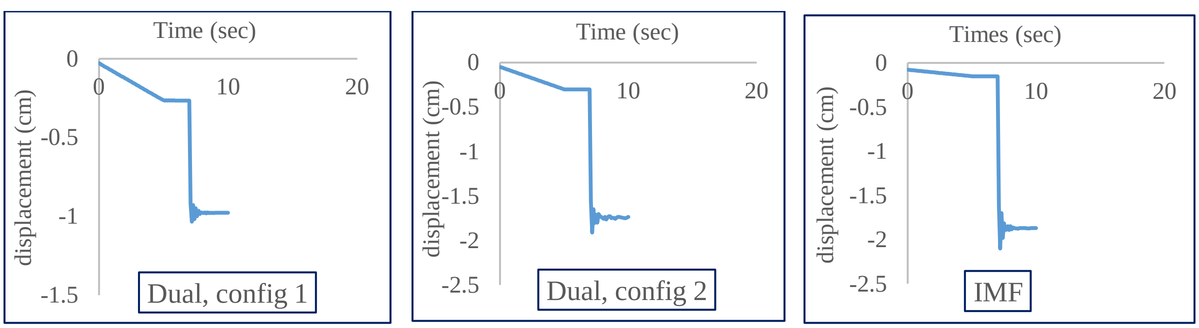


(c) Interior column loss

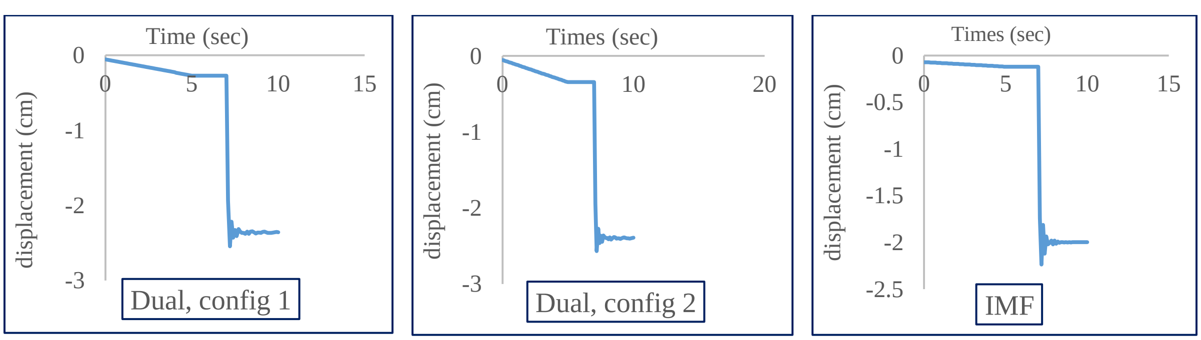
Fig. 12. Nonlinear dynamic analyses results of 10 story buildings; 1st story column loss.



(a) Corner column loss.

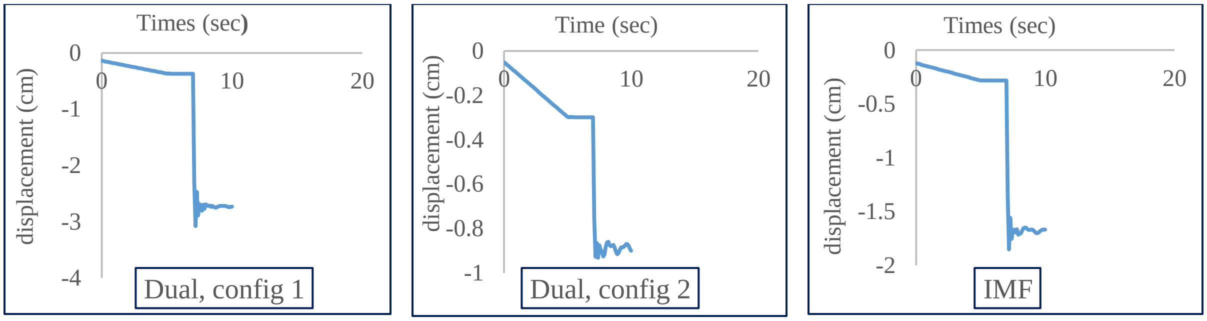


(b) Middle column loss

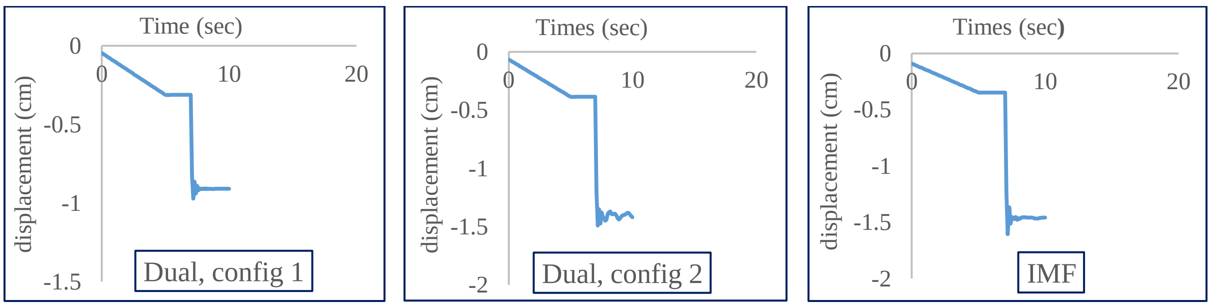


(c) Interior column loss

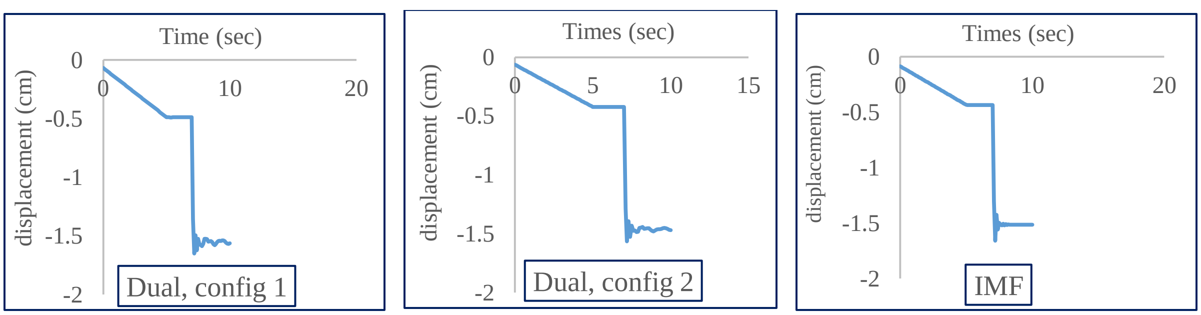
Fig. 13. Nonlinear dynamic analysis results of 4 story buildings; 3rd story column loss.



(a) Corner column loss.



(b) Middle column loss



(c) Interior column loss

Fig. 14. Nonlinear dynamic analysis results of 10 story buildings; 5th story column loss.

5. Conclusions

The main purpose of this study was to inquire the progressive collapse capacity of buckling-restrained braced frames as dual systems and to compare with conventional steel moment resisting frames. Nonlinear static and dynamic progressive collapse analyses were conducted on 4 and 10 story buildings. The alternate path method was employed and either first or mid-height story column was detached from the structural models. Pursuant to nonlinear static and dynamic analyses results, the code-based designed dual systems and IMFs have no potential of progressive collapse, and by removing either the corner, middle, and interior column, an alternate path is provided to absorb the forces as a result to column or adjacent BRBs loss.

The push-down analyses results indicate that progressive collapse load factors of dual systems are higher than those in IMFs except in one case. Dual system with configuration 1 and IMF have almost the same load factors when the corner column loss occurs, which can be associated to smaller beam and column sizes of dual systems in comparison to IMFs. In all of the dual and IMF structural models, the progressive collapse resisting capacity (i.e. load factor) increases as the building become taller. This occurs as a result of the increment of the structural elements which can absorb the column loss. Furthermore, in case of interior column loss, all structural systems reveal the same progressive collapse resisting capacity.

It is noteworthy to mention that despite the lower amount of steel material used in dual system in comparison to moment resisting frame, generally in most of cases, the progressive collapse resistance of the dual system is higher.

Based on dynamic analysis results, the amount of vertical displacement of the point above the removed column and the progressive collapse resistance, consequently, depends on location of column loss. The results indicate that in the dual system, the presence of BRBs above the detached column in the structural model (i.e. config.2) enhances the progressive collapse resistance of the system. The main outcomes of this study can be summarized as follow:

- When corner column and adjacent BRBs were removed in first and mid-height story of both 4 and 10 story buildings, dual system with configuration 2 possesses higher progressive collapse resistance in comparison to moment resisting frame. However, the dual system with configuration 1 exhibits lower progressive collapse resistance in comparison to moment resisting frame.
- When middle column and adjacent BRBs were removed in first and mid-height story of both 4 story and 10 story models, dual system with configuration 1 and 2 exhibit higher progressive collapse resistance in comparison to moment resisting frame, which can be associated to the catenary action of adjacent BRBs in removed column spans.
- When interior column and adjacent BRBs were removed in the first story of both models (i.e. 4 story and 10 story), dual systems with configuration 1 and 2 manifest higher progressive collapse resistance in comparison to IMF. When the interior column and adjacent BRBs in the third story of 4 story building is removed, IMF exhibits better progressive collapse response with lower top displacement in comparison to the dual systems. Moreover, when removing the interior column and adjacent BRBs in fifth story of 10 story building, the vertical displacement of top point in all systems are almost the same.

When column loss transpires in the mid-height story, the load factors decrease; however, they still remain above 1.0. In addition, all structural systems in all column removal scenarios have almost the same load factors. Pursuant to the results of nonlinear dynamic procedure, the amount of vertical displacement at the top point of mid-height story removed column is higher in comparison to the first story column loss case. As the number of stories increases, the top point vertical displacement decreases and the progressive collapse resistance of the building increases in both dual and IMF systems, as a result. It can be deducted that the progressive collapse response of dual systems is strongly dependent on the location of column loss and also the placement of the braces inside the structure. The presence of BRBs in the stories above the removed column significantly augments the progressive collapse resistance of the dual system as a result to the catenary action of the braces.

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